Appendix A: Hydraulic Design

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A.1 River Stage Frequency Plots

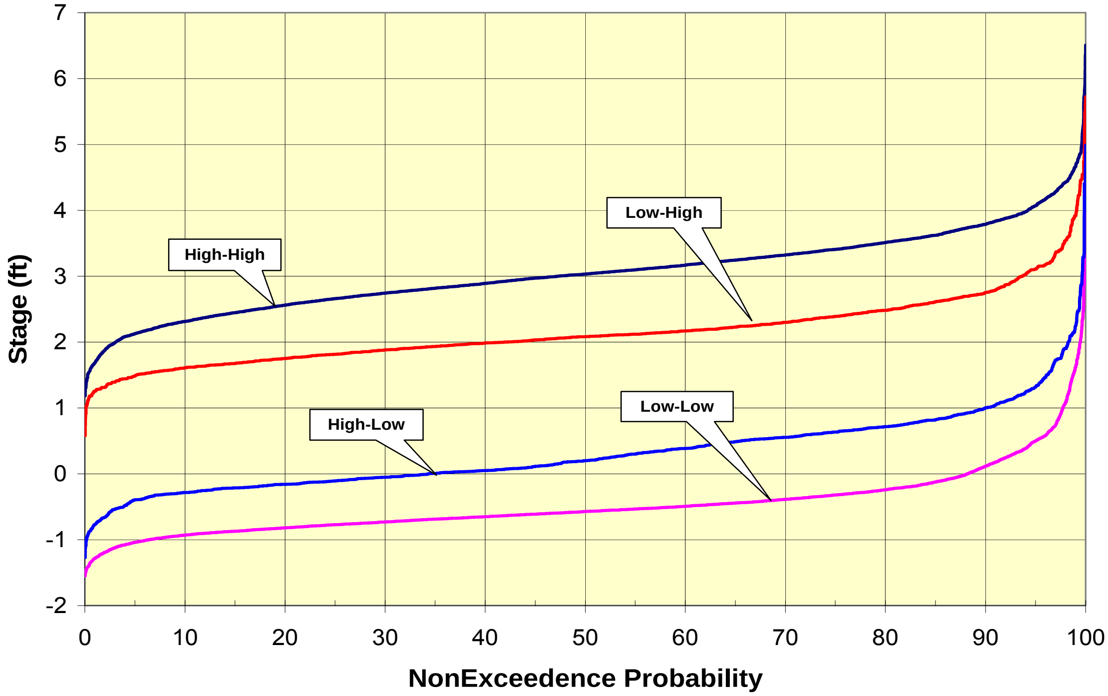

Figure A.1 – Cumulative frequency distribution curve of daily high-high and low-low stages for Webb Tract San Joaquin River Intake

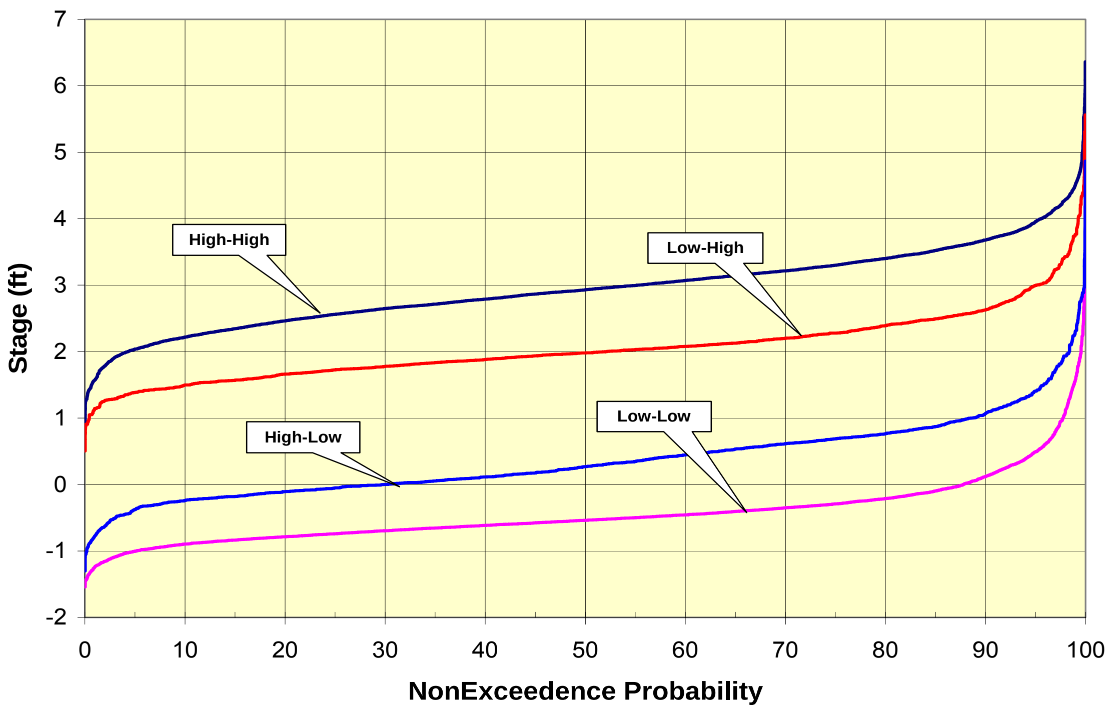

Figure A.2 – Cumulative frequency distribution curve of daily high-high and low-low stages for Webb Tract False River Intake

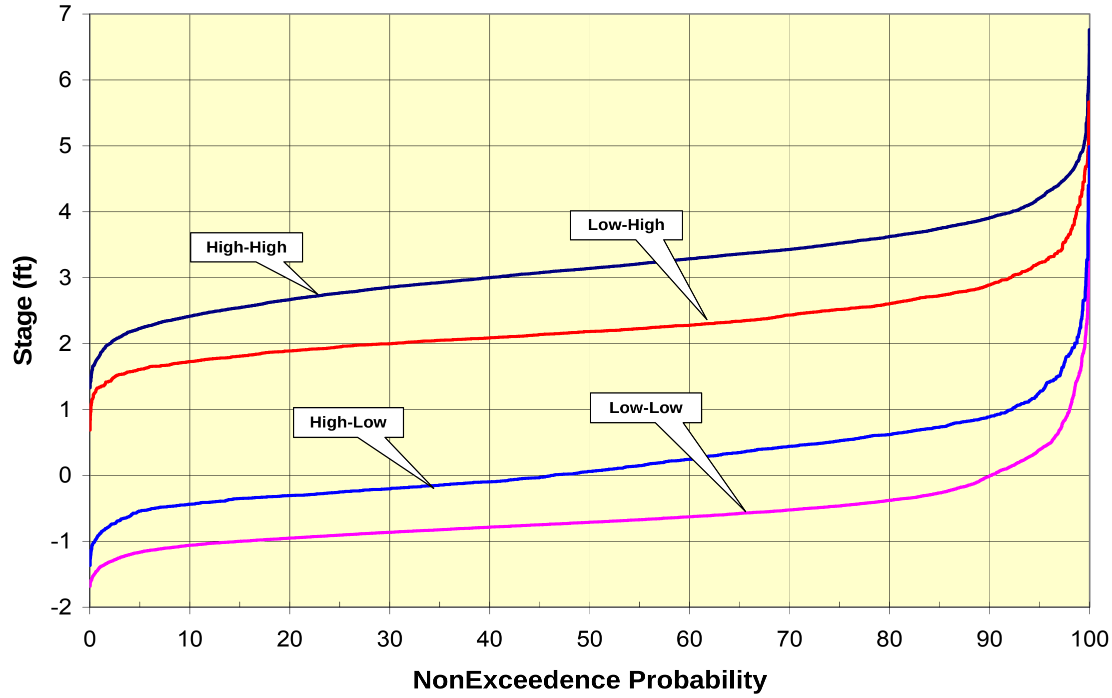

Figure A.3 – Cumulative frequency distribution curve of daily high-high and low-low stages for Bacon Island Middle River Intake

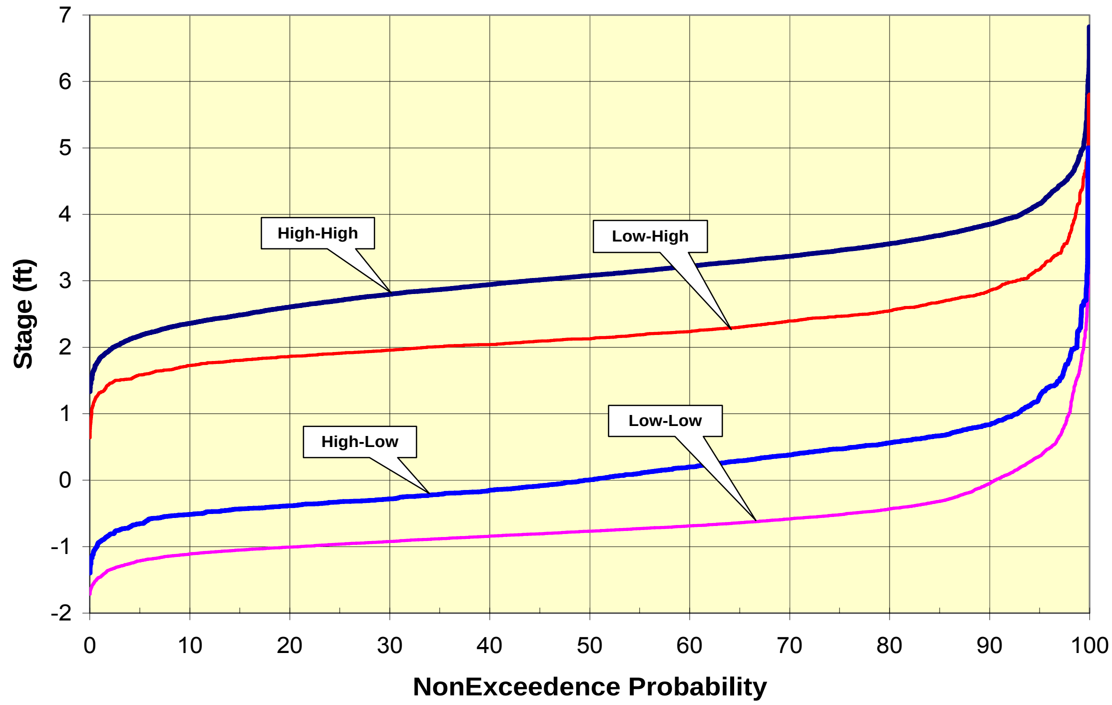

Figure A.4 – Cumulative frequency distribution curve of daily high-high and low-low stages for Bacon Island Santa Fe Cut Intake

A.2 Tail Water Depth Requirements for Intake Structures

Minimum tail water depth is the depth of water needed in the midbay of the integrated facility to dissipate the energy of high velocity water coming through Gate#1. A continuous sloping apron (3H:1V) provides a transition from the sill of Gate#1 to the floor of the midbay. A hydraulic jump is used to dissipate excess energy carried by the water entering the midbay. Minimum tail water depth is calculated as the sequent depth corresponding to the depth of water at the end of the sloping apron. First, a water surface profile was plotted for a design discharge of 2250 cfs and an apron width of 40 ft. Then the sequent depth was calculated corresponding to the depth of water at the end of the sloping apron. The sequent depth was then added to the midbay floor elevation to determine the minimum tail water elevation requirements for a hydraulic jump.

The sequent depth is calculated as:

$$d_2 = \tfrac{1}{2}\left(\sqrt{8F_1^2 + 1} - 1\right)d_1$$

where, d_1= the depth of water at the toe of the sloping apron,
F = Froude Number

Water Surface Profiles on the Slope

The depth of water on the sloping apron was determined using the direct step method. The energy equation between two sections separated by Δx could be written as

$$S_0\Delta x + y_1 + \frac{V_1^2}{2g} = y_2 + \frac{V_2^2}{2g} + S_f\Delta x$$

where,
S_o =Slope of the drop from Gate#1 sill to the bottom of the pool (3H:1V),
S_f =Slope of the energy line, using Manning's formula energy slope is given as

$$S_f = \frac{n^2V^2}{2.22R^{4/3}}$$

y =depth of flow,
Δx =distance between two sections,
V_1 =velocity at end 1 of the reach Δx,
V_2 =velocity at end 2 of the reach Δx

The water surface profile was determined from Gate #1 to the midbay. The water surface profile calculation assumes that the gate is discharging a maximum flow of 2250 cfs and the pool on the downstream side (midbay) is empty. Critical and normal depths were calculated for the gate sill section and the sloped section. An 'M3' profile was plotted from the gate opening to the end of the gate sill (or beginning of the slope). Since the horizontal length of the gate sill is very short (35ft), it was observed that the hydraulic jump does not form on the sill. An 'S2' profile was plotted from the beginning of the slope to the midbay floor level. The 'S2" profile was combined with the minimum tail water depth to generate the final water surface profile. Figures A.5 through A.8 show the critical depth, normal depth and water surface profiles for the integrated facilities.

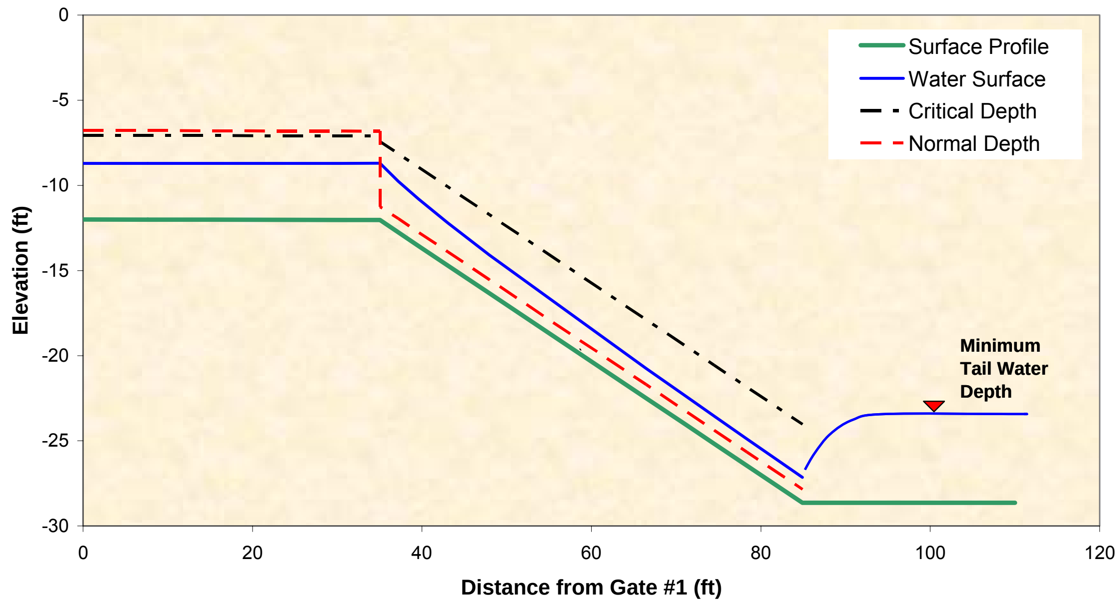

Figure A.5 – Water Surface Profile for Webb Tract San Joaquin River Integrated Facility

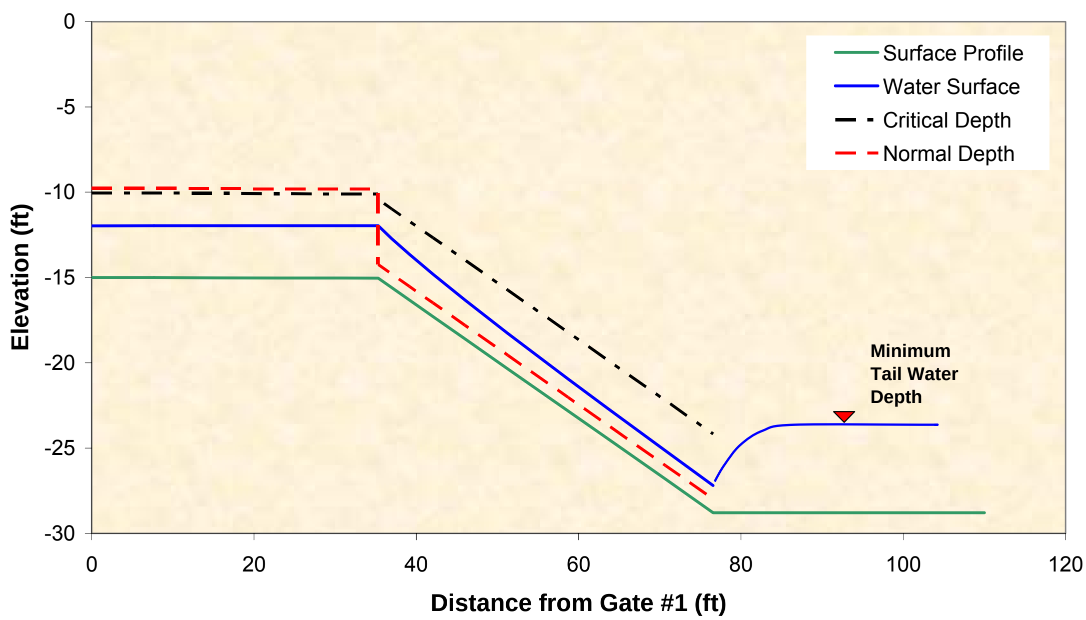

Figure A.6 – Water Surface Profile for Webb Tract False River Integrated Facility

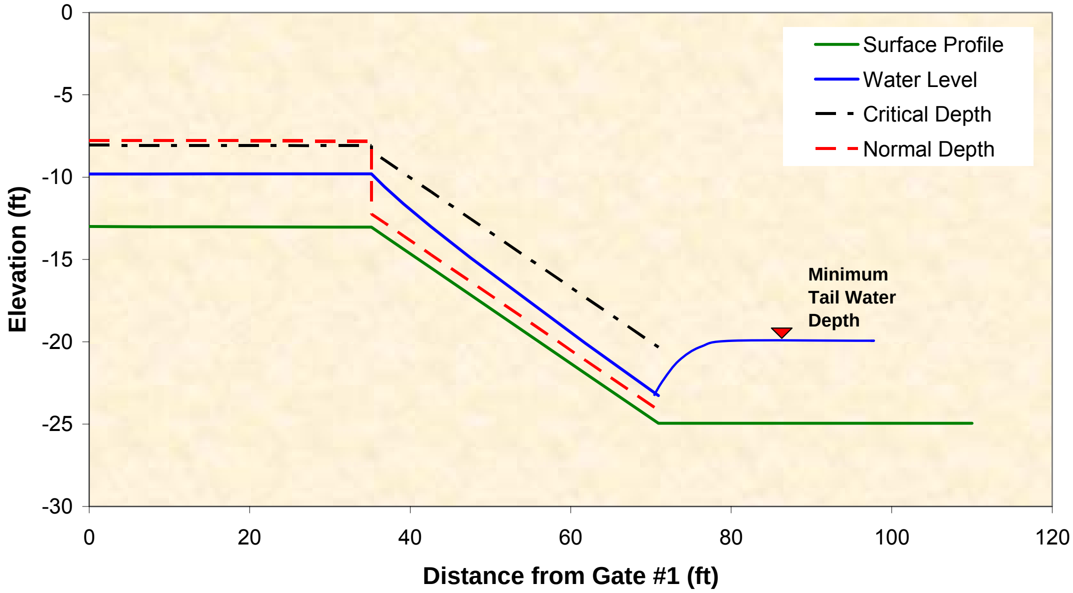

Figure A.7 – Water Surface Profile for Bacon Island Middle River Integrated Facility

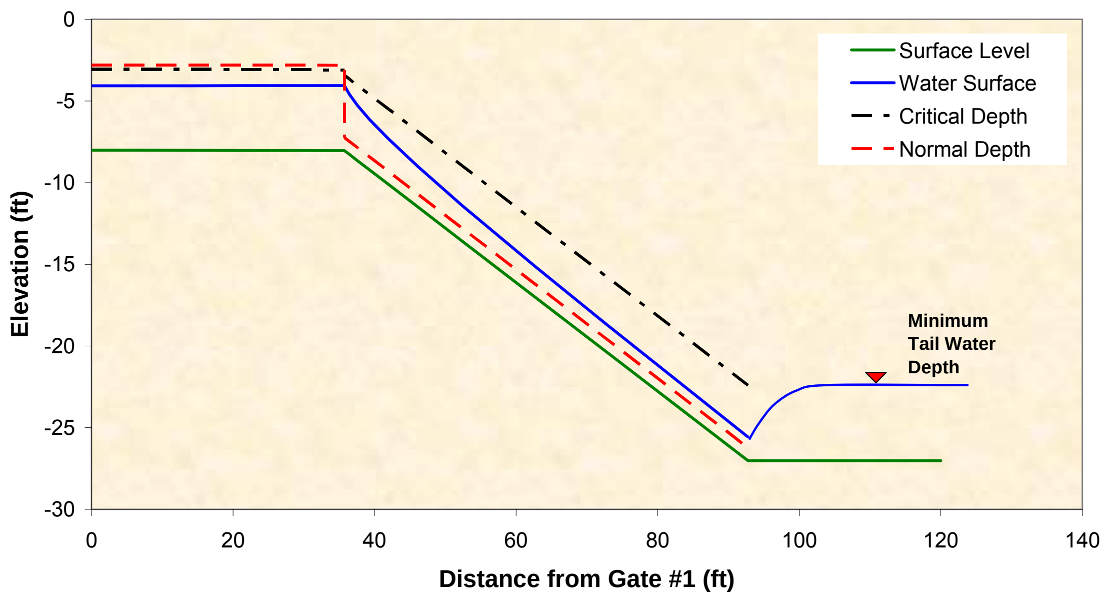

Figure A.8 – Water Surface Profile for Bacon Island Santa Fe Cut Integrated Facility

A.3 Flow Rating Curves

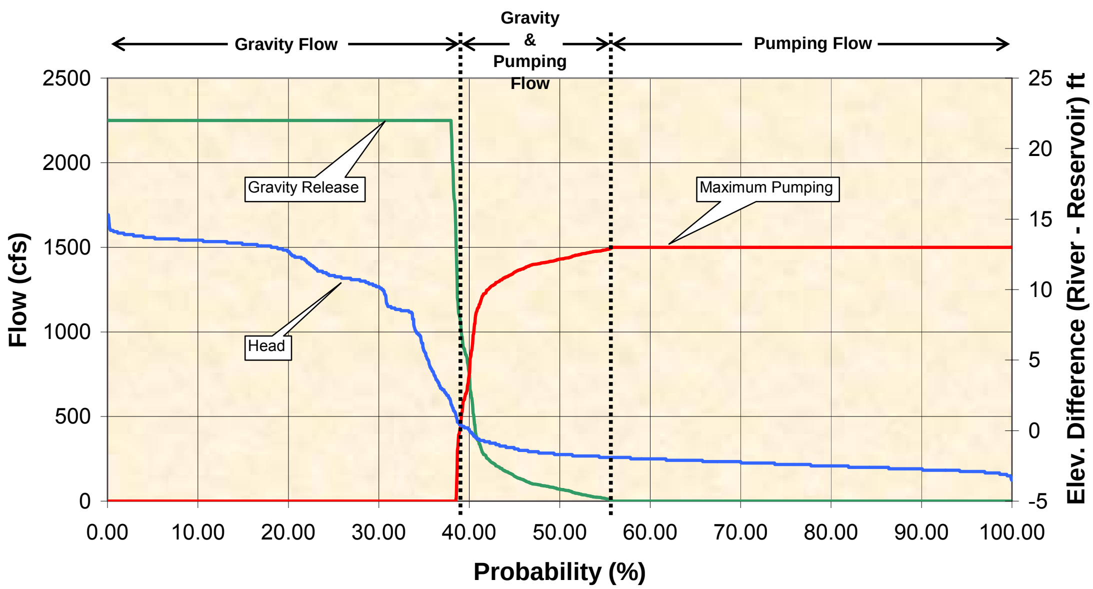

Figure A.9 – Inflow Rating Curve through Gate #1 for Webb Tract San Joaquin River Integrated Facility

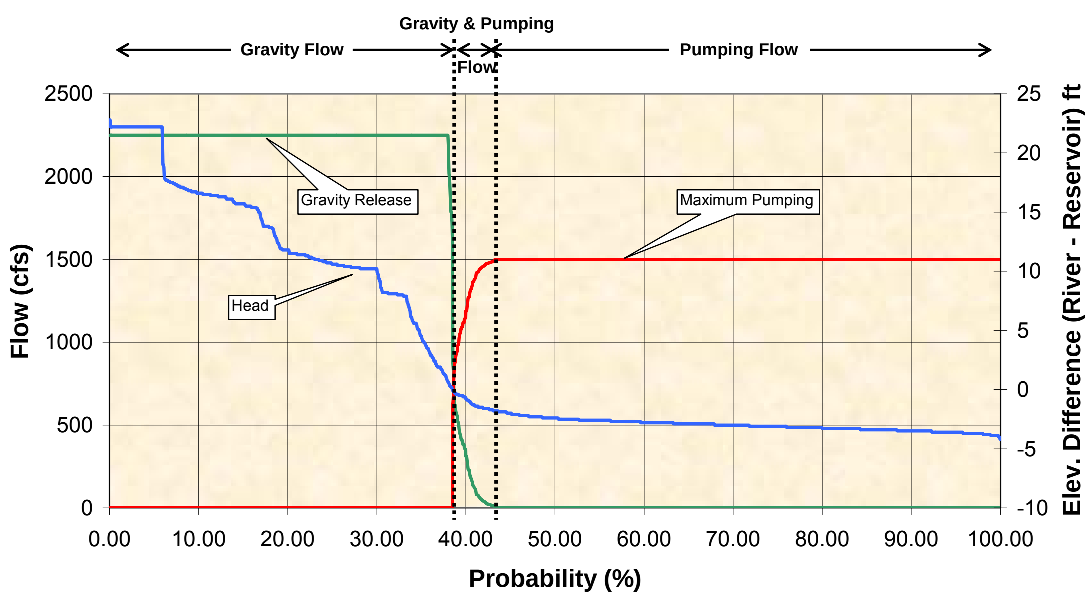

Figure A.10 – Inflow Rating Curve through Gate #2 for Webb Tract San Joaquin River Integrated Facility

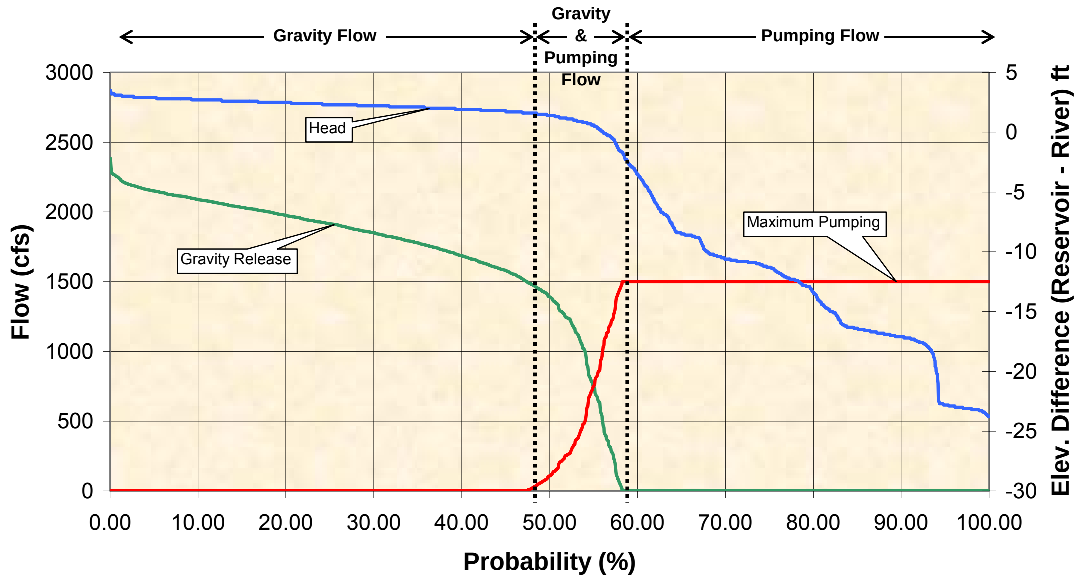

Figure A.11 – Outflow Rating Curve through Gate #3 for Webb Tract San Joaquin River Integrated Facility

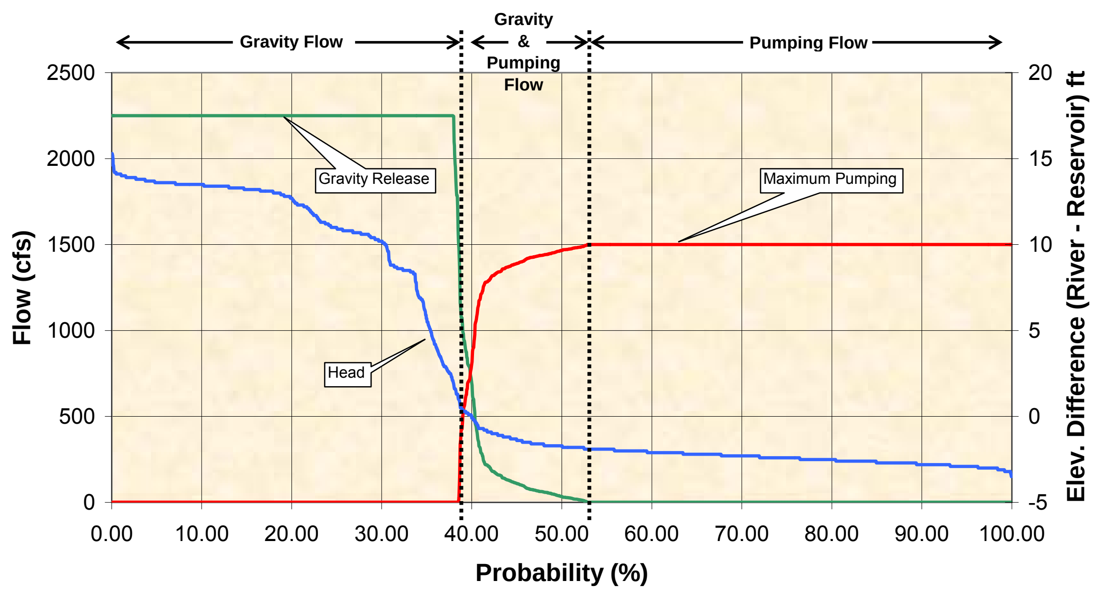

Figure A.12 – Inflow Rating Curve through Gate #1 for Webb Tract False River Integrated Facility

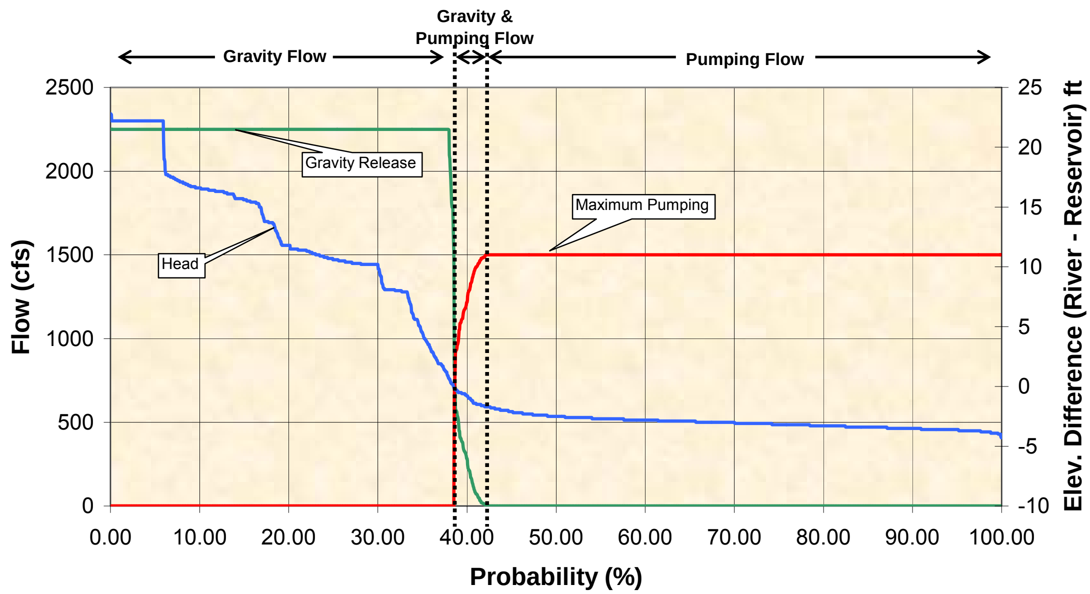

Figure A.13 – Inflow Rating Curve through Gate # 2 for Webb Tract False River Integrated Facility

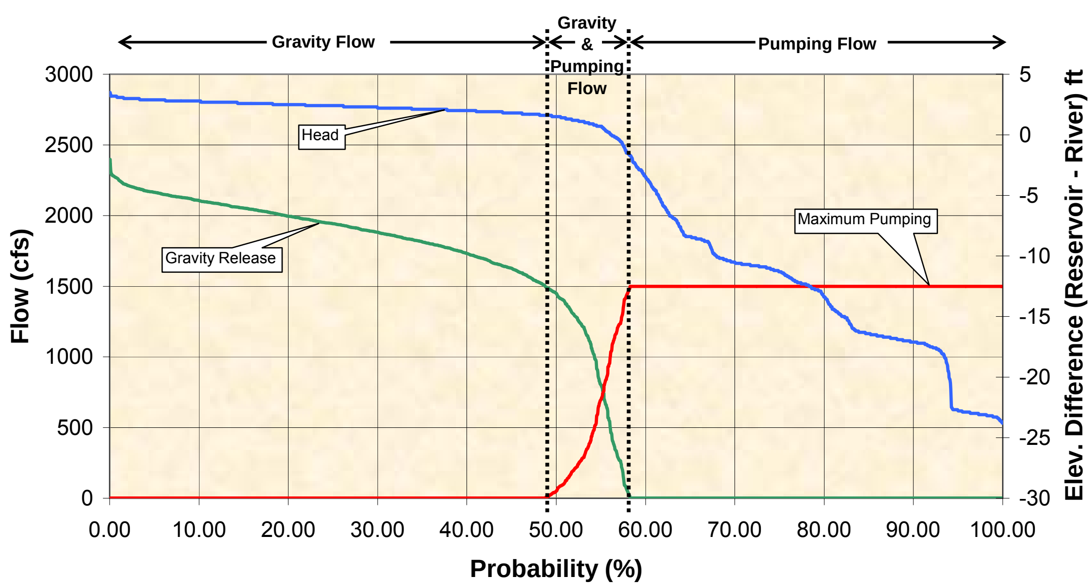

Figure A.14 – Outflow Rating Curve through Gate #3 for Webb Tract False River Integrated Facility

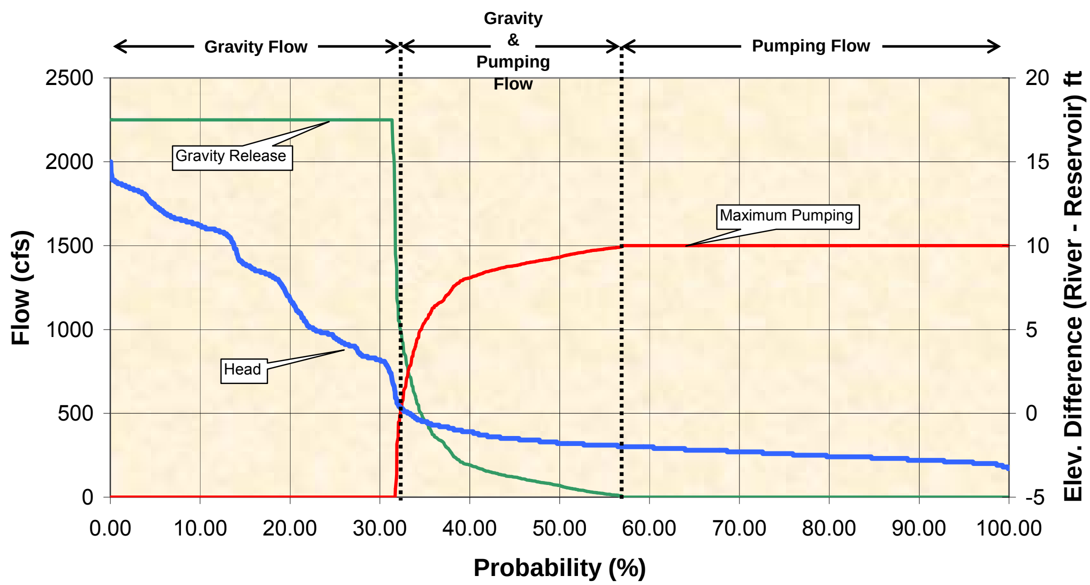

Figure A.15 – Inflow Rating Curve through Gate #1 for Bacon Island Middle River Integrated Facility

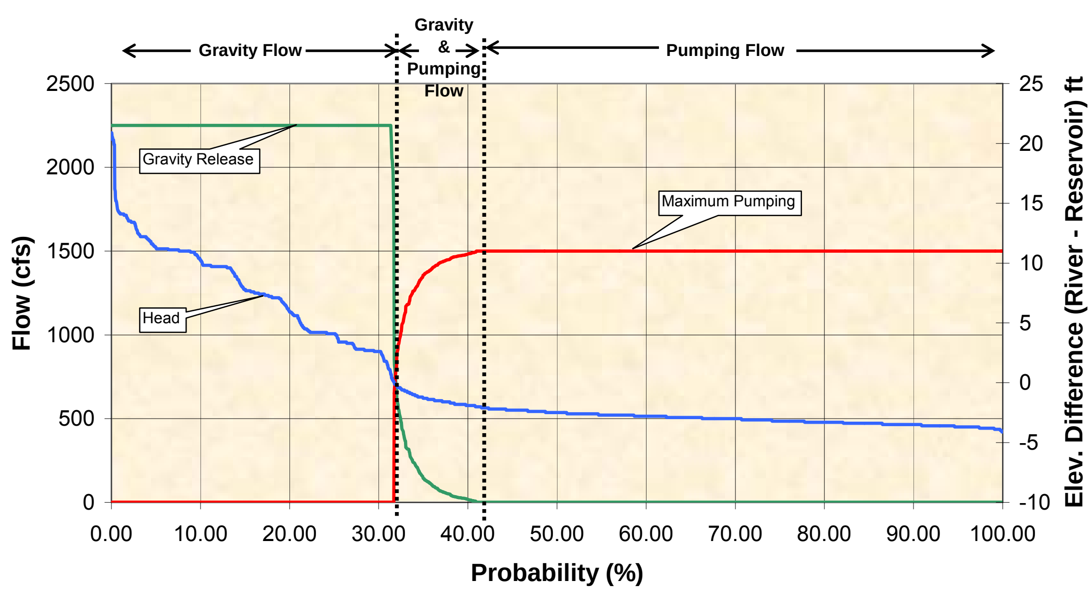

Figure A.16 – Inflow Rating Curve through Gate #2 for Bacon Island Middle River Integrated Facility

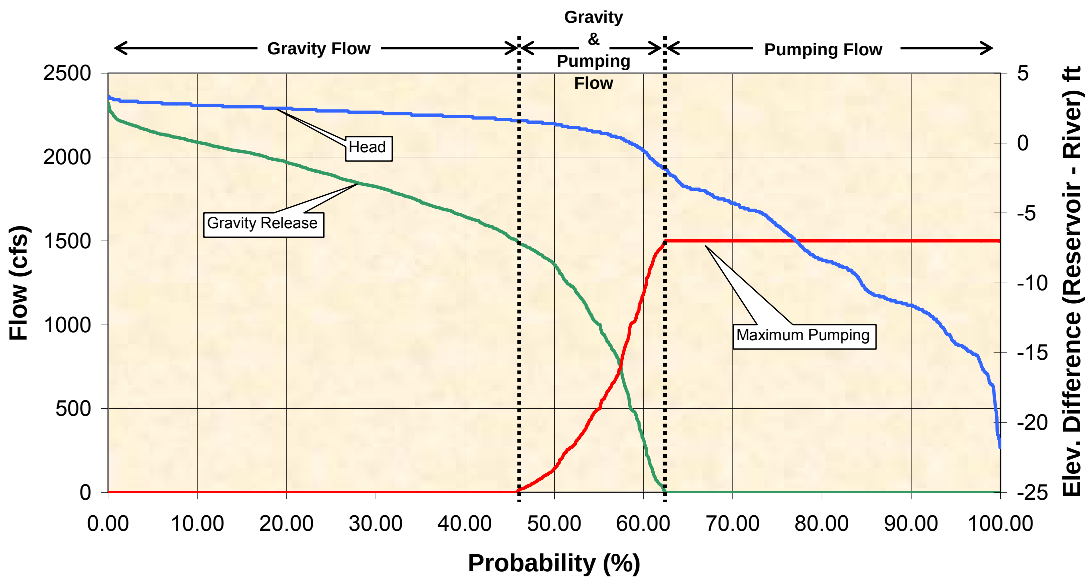

Figure A.17 – Outflow Rating Curve through Gate #3 for Bacon Island Middle River Integrated Facility

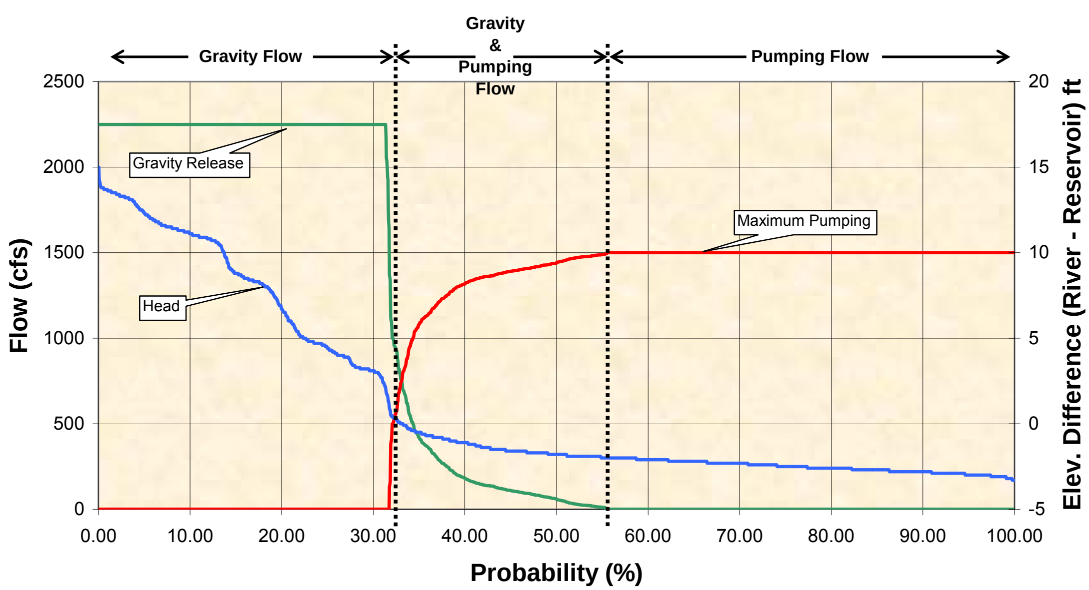

Figure A.18 – Inflow Rating Curve through Gate #1 for Bacon Island Santa Fe Cut Integrated Facility

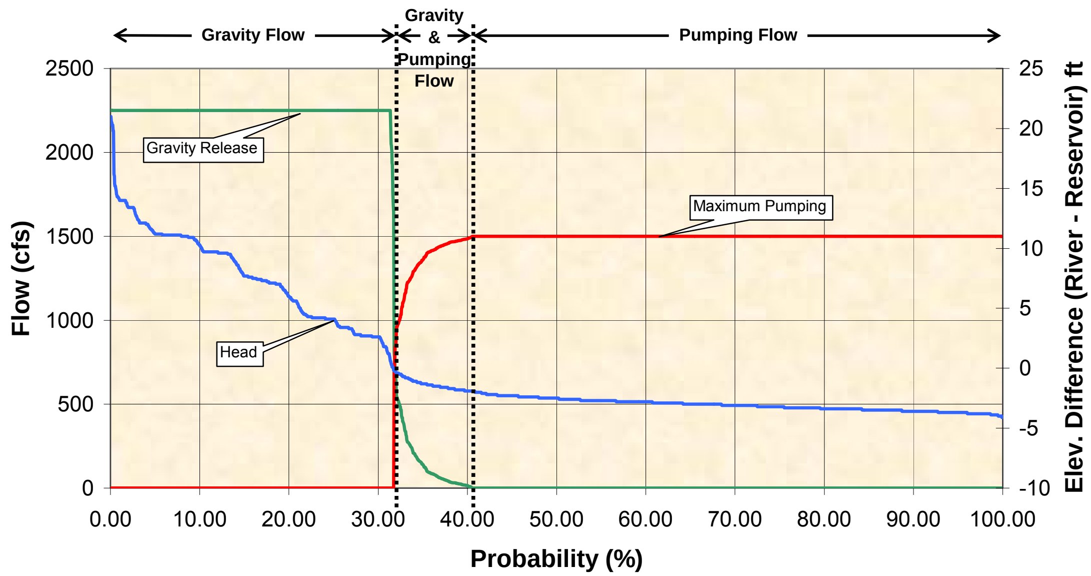

Figure A.19 – Inflow Rating Curve through Gate #2 for Bacon Island Santa Fe Cut Integrated Facility

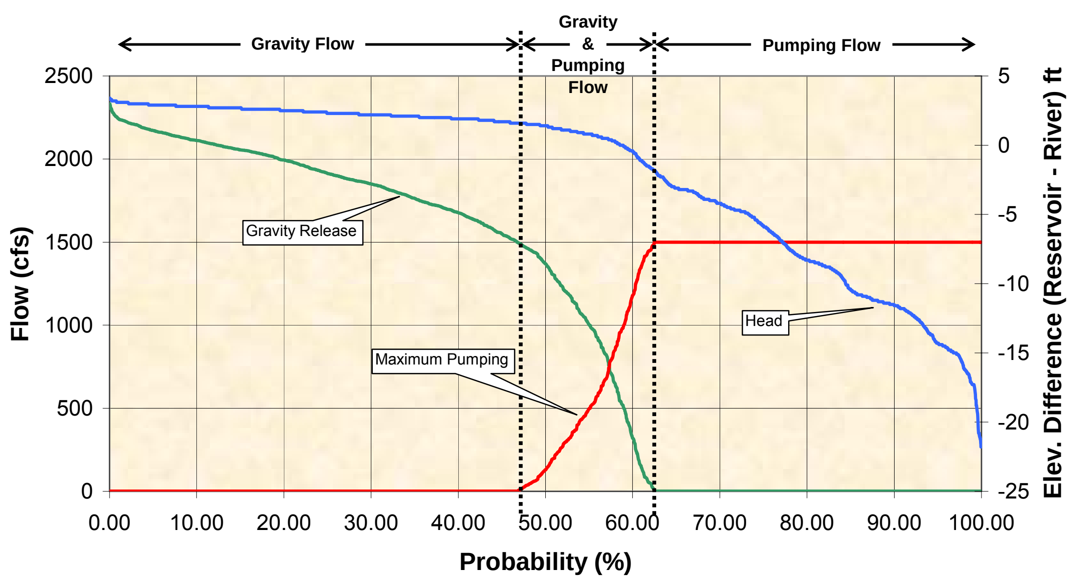

Figure A.20 – Outflow Rating Curve through Gate #3 for Bacon Island Santa Fe Cut Integrated Facility

A.4 Gate Design

A.4.1 Discharge Equation

The discharge through the gate opening was determined using the following equation:

$$Q = CA\sqrt{2gh}$$

where,
Q = discharge in cubic feet per second (cfs),
C = discharge coefficient. C for the Slide Gate and is assumed to be 0.60,
A = the area of the gate opening in square feet,
h = head available at the gate in feet and,
g = acceleration of gravity.

A.4.2 Gate Sizing Procedure

The required area of gate opening is calculated for a maximum flow of 2250 cfs through the gate and a maximum gate velocity of 8 fps.

Gate Area = Q/V ---------------------- (Equation 1)

Assuming a maximum discharge of 2250 cfs and exit velocity of 8 fps, required area for the gate is 281.25 sq. ft. Referring to 'Waterman Industries' Slide gate catalogues, three 12 feet by 10 feet gates give an opening of 360 sq feet. .

Head Requirement on the gate is checked for the area of gate opening calculated above using the orifice equation.

$$h = \frac{Q^2}{2gA^2C^2}$$ -------------------------- (Equation 2)

Solving equation 2 using the above input the required head is 1.69 ft.

Gate Discharge Rating Curve

Discharge Rating calculations were done for the Gate using maximum head difference across the gate. The maximum water surface elevation (WSEL) in the River at this site is +6.826 ft and the minimum WSEL at this site is -1.714 ft. The rating curve for the 12ft wide gate was plotted. Discharge through the gate was determined for a range of net head acting on the gate. Figure A.21 shows the rating curve for this 12 ft wide gate. Variation of Flow Velocity and Froude Number with the net head on the gate is shown in Figure A.22.

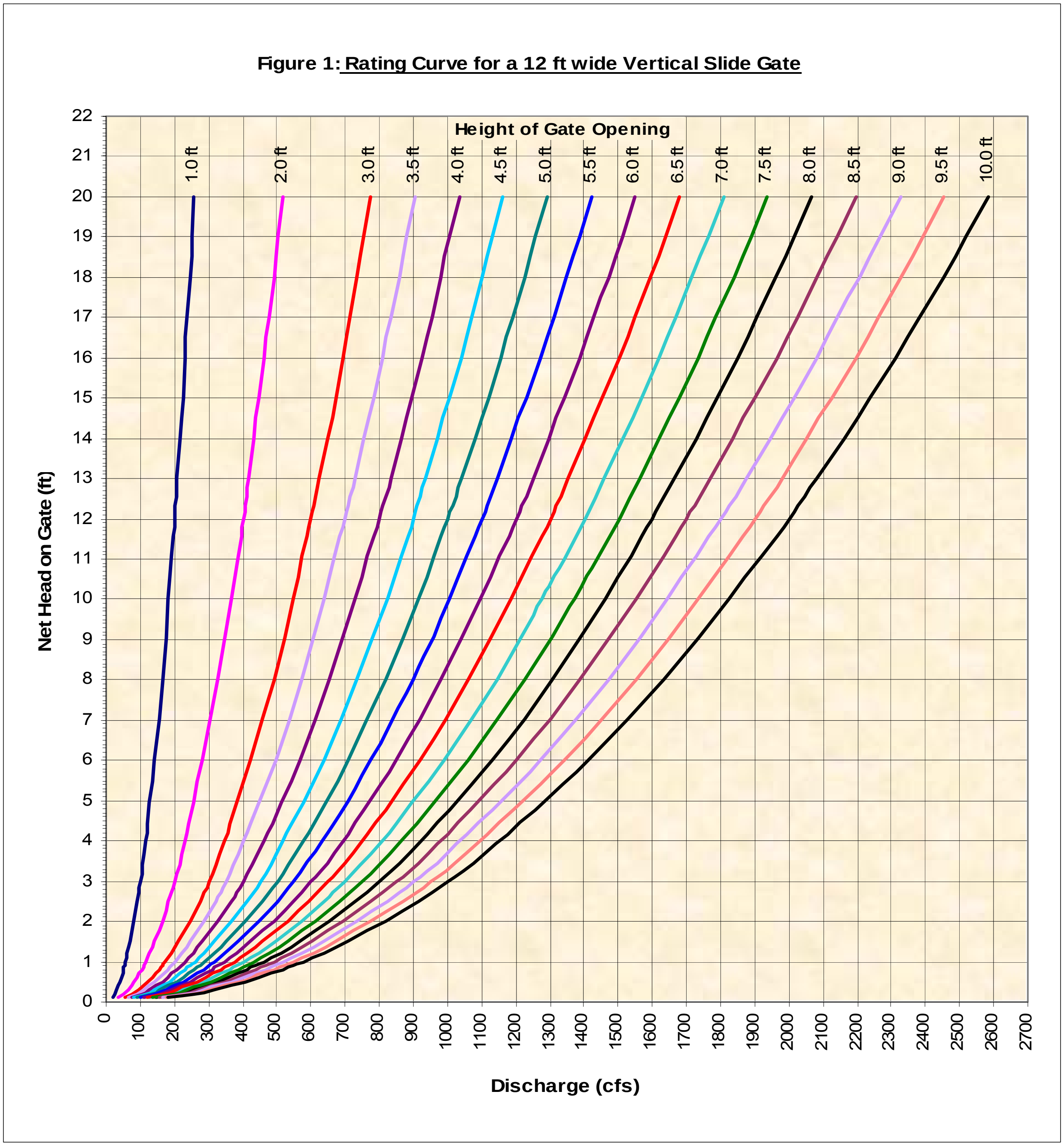

Figure A.21 – Flow Rating Curve for a 12 feet wide gate.

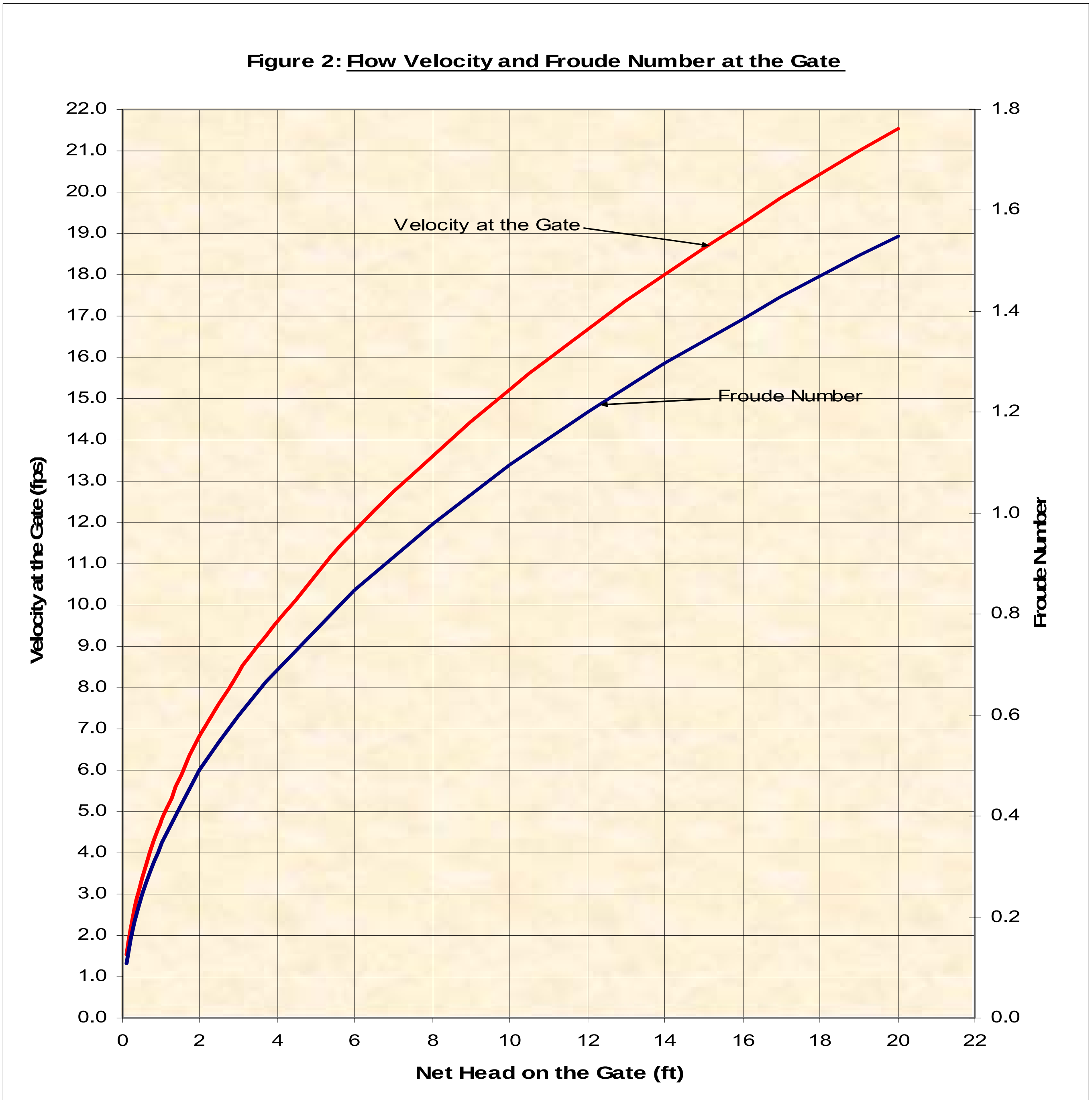

Figure A.22 – Froude Number and Velocity Variation (flow through a 12 ft wide gate)

A.5 Hydraulic Design Procedure for Pipe Conduit

The design procedure for this analysis involves selecting a combination of pipe sizes that will carry the design discharge via gravity flow from the reservoir to the river (bypass channel). The water levels in both reservoir and the river will fluctuate, so the available head will vary.

Given the variation of available head between the reservoir and the river, gravity flow capacity through the conduit pipes was determined by the energy balance approach. The capacity calculations include pipe friction losses, calculated using the Darcy-Weisbach formula and minor head losses (such as entrance, exit, valves, fittings, contractions, expansions, etc.). The hydraulic design methodologies, formulas and procedures used are as follows.

Formulas used for Gravity Flow Calculations

Energy Balance Equation

$$Z_1 + \frac{p_1}{\gamma} + \frac{V_1^{\ 2}}{2g} = Z_2 + \frac{p_2}{\gamma} + \frac{V_2^{\ 2}}{2g} + h_{loss} \qquad \text{Eqn. 1}$$

Total Head Loss

$$h_{loss} = h_f + h_{\min or} \qquad \text{Eqn. 2}$$

Darcy-Weisbach Head Loss Formula

$$h_f = \frac{fL}{d}\frac{V^2}{2g} \qquad \text{Eqn. 3}$$

where,

f = Darcy friction factor
L = Length of pipe, ft
d = Internal pipe diameter, ft
V = Average velocity of flow in the pipe, ft/sec
n = Manning's roughness coefficient
g = Acceleration due to gravity, ft/sec^2

Minor Head Loss

$$h_{\min or} = \frac{V^2}{2g}\left(K_{entrance} + K_{valves} + K_{fittings} + K_{contraction} + K_{enl\arg ment} + K_{exit}\right) \text{ Eqn. 4}$$

K = Loss coefficient

Simplified, the energy equation can be written in terms of the head losses and water levels, Z_1 and Z_2, such that

$$Z_1 - Z_2 = \frac{V^2}{2g}\left(\frac{fL}{d} + K_{entrance} + K_{valves} + K_{fittings} + K_{contraction} + K_{enl\arg ment} + K_{exit}\right)$$

Eqn. 5

Equation 5 in terms of Flow

$$Q = \sqrt{\frac{(Z_1 - Z_2)2g\left(\frac{\Pi d^2}{4}\right)^2}{\frac{fL}{d} + \sum K}} \qquad \text{Eqn. 6}$$

Reynolds Number, Re

$$\mathrm{Re} = \frac{Vd}{\nu} \text{ Eqn. 7}$$

where,

V = Average velocity in ft/sec
d = Internal pipe diameter in feet

v = Kinematic viscosity of water in ft^2/sec

Darcy Friction Factor Formula (Jain 1976)

$$\frac{1}{\sqrt{f}} = -2\log_{10}\left(\frac{\in}{3.7d} + \frac{5.72}{\mathrm{Re}^{0.9}}\right) \quad \text{Eqn. 8}$$

where,
$\in$ = Equivalent roughness in feet
d = Internal pipe diameter in feet
Re = Reynolds number

Design Procedure

To calculate gravity flow capacity, where velocity is not known, the following trial and error procedure was followed:

- Assume a value of the Darcy friction factor, f ,based on the pipe size, material, and equivalent roughness($\in$)
- Calculate the flow in the pipe using Eqn. 6 above with the assumed Darcy friction factor
- Calculate the velocity in the pipe based on the flow computed in Step 2
- Calculate the Reynolds Number in the pipe using Eqn. 7 above
- Calculate a revised Darcy friction factor, f , based on the Reynolds number computed in Step 4 and Eqn. 8 above
- Calculate the revised flow in the pipe using Eqn. 6 above with the revised Darcy friction factor
- Calculate the revised velocity in the pipe based on the flow computed in Step 6
- Calculate the revised Reynolds Number in the pipe using Eqn. 7 above
- Calculate a second revised Darcy friction factor, f , based on the Reynolds number computed in Step 8 and Eqn. 8 above
- Compare the revised Darcy friction factors computed in Steps 5 and 9. If f stabilizes, then calculate the flow in the pipe based on the stabilized Darcy friction factor.

The spreadsheet procedure used to calculate gravity flow capacity as described above is shown in Figures A.23 and A.24. Figure A.25 shows gravity flow rating curves for the two 8-ft conduit pipes, for the 6-ft conduit pipe, and for all three conduit pipes combined.

STEP 1	
All Green Area's are User Inputs	
	USER INPUTS
Temperature (F)	60
Kinematic Viscosity (ft^2/sec)	1.217E-05
Equivalent Roughness, ε (ft)	0.002
Gravitational Constant, g (ft/sec^2)	32.2
Pi, Π	3.14
Pipe Diameter, d (ft)	8
Pipe Length, (ft)	544
Total Head Loss Coefficient	3.5
Manning's "n"	0.013
Hazen-Williams 'C' - Low	100
Hazen-Williams 'C' - High	140

			STEP 2 Input an Initial Friction Factor				STEP 3 -- Check to see if the following statement is true: "Darcy's Friction Factor Converges, So Use Revised Flow"	Use Revised Flow			
Reservoir Elevation	River Elevation	Total Head	TRY Darcy Friction Factor	INITIAL Darcy Flow	INITIAL Darcy Velocity	INITIAL Reynolds Number	REVISED 1 Darcy Friction Factor	REVISED Darcy Flow	REVISED Darcy Velocity	REVISED Reynolds Number	REVISED 2 Darcy Friction Factor
Z_1	Z_2	H	f	Q	V	Re	f	Q	V	Re	f
4	4	0	0.016	0	0.00	0.00E+00	0.00000	0	0.00	0.00E+00	0.00000
4	3.5	0.5	0.016	133	2.65	1.74E+06	0.01495	121	2.42	1.59E+06	0.01500
4	3	1	0.016	188	3.75	2.46E+06	0.01480	172	3.42	2.25E+06	0.01484
4	2.5	1.5	0.016	231	4.59	3.02E+06	0.01473	211	4.19	2.75E+06	0.01476
4	2	2	0.016	266	5.30	3.48E+06	0.01469	243	4.84	3.18E+06	0.01472
4	1.5	2.5	0.016	298	5.92	3.89E+06	0.01466	272	5.41	3.56E+06	0.01469
4	1	3	0.016	326	6.49	4.27E+06	0.01464	298	5.93	3.90E+06	0.01466
4	0.5	3.5	0.016	352	7.01	4.61E+06	0.01462	322	6.40	4.21E+06	0.01464
4	0	4	0.016	377	7.49	4.93E+06	0.01461	344	6.85	4.50E+06	0.01463
4	-0.5	4.5	0.016	399	7.95	5.22E+06	0.01460	365	7.26	4.77E+06	0.01462
4	-1	5	0.016	421	8.38	5.51E+06	0.01459	385	7.66	5.03E+06	0.01460
4	-1.5	5.5	0.016	442	8.79	5.78E+06	0.01458	404	8.03	5.28E+06	0.01460

Figure A.23 – Spreadsheet Procedure Used to Calculate Gravity Flow Capacity in 8 foot Diameter Conduit Pipe

STEP 1

All Green Area's are User Inputs

	USER INPUTS
Temperature (F)	60
Kinematic Viscosity (ft^2/sec)	1.217E-05
Equivalent Roughness, ε (ft)	0.002
Gravitational Constant, g (ft/sec^2)	32.2
Pi, Π	3.14
Pipe Diameter, d (ft)	6
Pipe Length, (ft)	544
Total Head Loss Coefficient	3.5
Manning's "n"	0.013
Hazen-Williams 'C' - Low	100
Hazen-Williams 'C' - High	140

STEP 2 **Input an Initial Friction Factor**

STEP 3 -- Check to see if the following statement is true:

"Darcy's Friction Factor Converges, So Use Revised Flow"

Use Revised Flow

Reservoir Elevation	*River Elevation*	*Total Head*	*TRY Darcy Friction Factor*	*INITIAL Darcy Flow*	*INITIAL Darcy Velocity*	*INITIAL Reynolds Number*	*REVISED 1 Darcy Friction Factor*	*REVISED Darcy Flow*	*REVISED Darcy Velocity*	*REVISED Reynolds Number*	*REVISED 2 Darcy Friction Factor*
Z_1	Z_2	H	f	Q	V	Re	f	Q	V	Re	f
4	4	0	0.016	0	0.00	0.00E+00	0.00000	0	0.00	0.00E+00	0.00000
4	3.5	0.5	0.016	72	2.55	1.26E+06	0.01591	66	2.33	1.15E+06	0.01596
4	3	1	0.016	102	3.61	1.78E+06	0.01575	93	3.30	1.63E+06	0.01579
4	2.5	1.5	0.016	125	4.42	2.18E+06	0.01567	114	4.04	1.99E+06	0.01570
4	2	2	0.016	144	5.10	2.51E+06	0.01563	132	4.67	2.30E+06	0.01565
4	1.5	2.5	0.016	161	5.70	2.81E+06	0.01559	148	5.22	2.57E+06	0.01562
4	1	3	0.016	177	6.25	3.08E+06	0.01557	162	5.72	2.82E+06	0.01559
4	0.5	3.5	0.016	191	6.75	3.33E+06	0.01555	175	6.18	3.04E+06	0.01557
4	0	4	0.016	204	7.21	3.56E+06	0.01553	187	6.60	3.26E+06	0.01556
4	-0.5	4.5	0.016	216	7.65	3.77E+06	0.01552	198	7.00	3.45E+06	0.01554
4	-1	5	0.016	228	8.06	3.98E+06	0.01551	209	7.38	3.64E+06	0.01553
4	-1.5	5.5	0.016	239	8.46	4.17E+06	0.01550	219	7.74	3.82E+06	0.01552

Figure A.24 – Spreadsheet Procedure Used to Calculate Gravity Flow Capacity in 6 foot Diameter Conduit Pipe

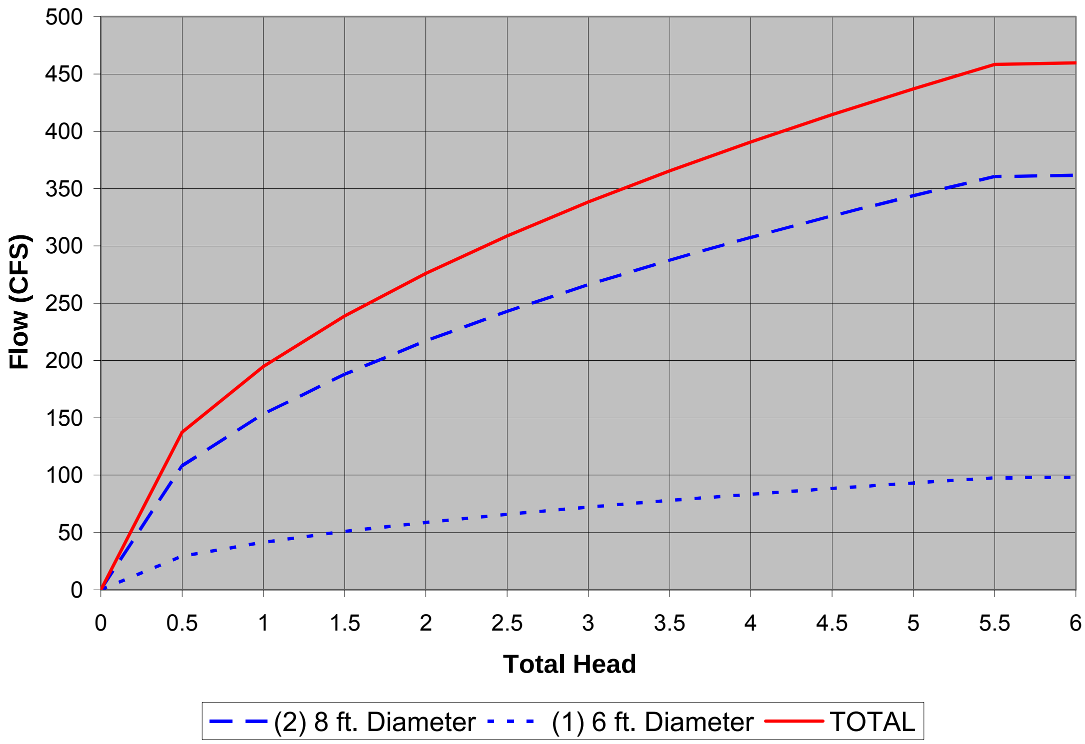

Figure A.25 – Gravity Flow Rating Curve through the Conduit Pipes

A.6 D-Load Strength

The 0.01-inch crack D-load strength ($D_{0.01}$) is the maximum three-edge bearing-test load supported by a concrete pipe before a crack occurs having a width of one one-hundredth (0.01) of an inch measured at close intervals throughout the length of at least 1 foot. The $D_{0.01}$ strength is determined using the following expression.

$$D_{0.01} = \left(\frac{W_e}{L_e} + \frac{W_l}{L_l} \right) \frac{1}{D}$$

where

W_e = Dead load due to earth cover (lbs/ft)

W_l = Live load due to surface surcharge (lbs/ft)

L_e = Load factor for earth load based upon class of bedding selected

L_l = Load factor for live load

D = Internal diameter of the pipe (ft)

A.7 Conduit Pipe Outlet Energy Dissipater

Design Procedure

- Capacity: Discharge, q, per foot of width of baffled apron of 32 cfs was selected from the tabulated baffled apron dimensions in 'Design of Canals and Related Structures' by USBR for the capacity of 1500 cfs.
- Inlet: Chute width of 48 feet was chosen and it falls within the range recommended in the above Bureau publication rectangular inlet section.
- Sill Control: The inlet sill length should be at least $2d_1$. The required height of the sill above the inlet floor was determined from the energy balance between the apron inlet and the flow at the conduit outlet.

  Thus,

  $$E_{s_1} = E_{s_c} + h_i + h_s$$

  or

  $$h_s = E_{s_1} - E_{s_c} - h_i$$

  where,

  h_s = height of the sill,

  $E_{s1} = d_1 + h_{v1}$ in the upstream channel,

  $E_{sc} = d_c + h_{vc}$ in the control section at the sill,

  h_i = inlet loss

  $= 0.5\Delta h_v$

  $= 0.5(h_{vc} - h_{v1})$

  $$= 0.5\left[\frac{V_c^2}{2g} - \frac{V_1^2}{2g} \right]$$

The curvature of the sill crest terminates at its point of tangency with the slope of the downstream apron. This point should not be more than 12 inches in elevation below the crest. This was assured by limiting the radius of curvature to a maximum of 9 feet. A 3 foot radius was used.

Baffled Apron Dimensions

- Slope**:** Slope of the chute floor and side walls was set at 2 to 1 (same as that of the levee).
- First row of baffles: The first row of baffles was set so that the base of the upstream face is at the downstream end of the invert curve and no more than 12 inches in elevation below the crest.
- Baffle block height: Baffle block height, h_b, should be about 0.9 times critical depth, dc, to nearest inch.
- Baffle block widths and spaces: Baffle block widths and spaces should be equal, and not less than h_b, but not more than 1-1/2 h_b. Partial blocks, having a width not less than $1/3h_b$ and not more than 2/3 h_b should be placed against the sidewalls in rows 1,3,5 etc. Alternate rows of baffle blocks were staggered so that each block is downstream from a space in the adjacent row.
- Slope distance between baffle blocks: The slope distance, s, between rows of baffle blocks, should be at least $2h_b$, but no greater than 6 ft. A spacing of 6ft was used for all blocks.
- Minimum rows of baffle blocks: A minimum of four rows of baffle blocks should be used. The baffle apron was extended so that the top of at least one row of baffle blocks will be below the bottom grade of the outlet channel. The apron should be extended beyond the last row of blocks a distance equal to the clear space between block rows.
- Longitudinal thickness of Baffle Blocks: Baffle blocks are constructed with their upstream faces normal to the chute floor. The longitudinal thickness, T, of the baffle blocks at the top should be at least 8 inches, but not more than 10 inches. Longitudinal thickness of 10 inches was used.
- Height of walls: Height of walls to provide adequate freeboard is 3 times the baffle block height measured normal to the chute floor.

Figure A.26 shows the spread sheet used to determine the dimension of different components of the baffled apron energy dissipater.

CONDUIT OUTLET DESIGN ON RESERVOIR SIDE:

BAFFLED APRON DROP DESIGN

(Webb Tract, San Joaquin River Integrated Facility)

Step 1: **Hydraulic Properties at Pipe Outlet:**

	Input	Input	Input	Input	Input		
	Maximum Discharge (cfs) for an 8ft diameter	Diameter of the pipe, (ft)	Elevation of Invert, (ft)	Discharge Required per foot of Chute width from table (cfs/ft)	Elevation of Outlet Channel Invert, (ft)	Drop in Invert Elevation, (ft)	Width of chute drop, (ft)
	Q	**D**		**q**			$B = \frac{Q}{q}$
Channel Properties at 1	1500.00	8.00	-12.00	50 to 60	-18.00	6.00	46.88
Use Chute drop width,B=	48.00	(Input)					
Use Discharge per foot, q=	32	(Input)					

Step 2: **Limits of baffle block dimensions, based upon critical depth, dc:**

Critical Depth, dc=	3.16	For a rectangular channel, critical depth is given b
Block Height, h_b=0.9*d_c=	2.85	where, q=recommended discharge/ft
Minimum Block width & space, w_{min}=h_b=	2.85	Use w = 4.00
Maximum Block width & space, w_{max}=1.5*h_b=	4.27	

$$d_c = \sqrt[3]{\frac{q^2}{g}}$$

Step 3: **Exact dimensions of baffle blocks and chute width as partial block width:**

width & space, wp=(1/3)*h_b=	0.95	Use w_p= 1.00
width & space, wp=(2/3)*h_b=	1.90	

Then use alternate rows as follows:

Rows 1 and 3:			Rows 2 and 4:		
5 full blocks=	5*w	20	6 full blocks=	6*w	24
6 full spaces=	6*w	24	5 full space=	5*w	20
2 half blocks=	1*w	4	2 half spaces=	1*w	4
B=	12*w	48	B=	12*w	48
B=	48		B=	48	

Step 4: **Recalculating the height of blocks, h_b:**

First calculated the discharge per foot, q, for total capacity of 1500 cfs. Then using this q, calculated critical depth, d_c and then height of block, h_b=.9 d_c

$$d_c = \sqrt[3]{\frac{q^2}{g}}$$

q=	31.25	cfs/ft
dc=	3.0832488	ft
hb=	2.77	ft

Use Top width of block= 10 inches (See Reference 1, Page 303)

Step 5: **Inlet length, L_1:**

Depth of flow, d_1, in the rectangular section of base width, 48 ft, just after the flared section of pipe outlet is calculated using King's Method. A mild slope of .001 was assumed.

d_1=	3.7	<--Input	(n=.012)
L_1= 2*d_1=	7.4		

Step 6: **Inlet sill height, h_s:**

Assumed that critical depth occurs at the sill.

Then, $h_s = E_{s1} - E_{sc}$ - inlet losses

	Discharge , Q (cfs)	Width of rectangular drop apron, b (ft)	Depth of water surface, d1 and dc	Area at (1), A_1 (f^{t2})	Velocity, V (ft/s)	Velocity Head, h_v (ft)	Specific Energy, (ft)	Inlet Loss	Height of Sill, h_s (ft)
Channel Properties at pipe outlet(at the end of flared section)	1500.00	48.00	3.70	177.60	8.45	1.11	4.81	-0.20	0.33
Channel Properties at beginning of apron drop	1500.00	48.00	3.16	151.90	9.87	1.51	4.68		

Use h_s = 4 inches

Step 7: **Checked Inlet Velocity to minimize splashing:**

Depth at inlet cutoff, d_1= $h_s + d_c + h_{vc}$= 5.01 Enterance Velocity (fps)= 6.24

Critical Velocity over crest= 9.87

Inlet velocity is a little less than half of the critical velocity, so splashing will be minimized.

Step 8: **Sill length, L_2, and dimension e :**

Slope of Invert = 1 2 (Input)

θ	R	z=tan(θ/2)*R	y=sinθ*z	x=y/tanθ	L_2=x + z	e = hs-y
26.57	3.00	0.71	0.32	0.63	1.34	0.02

Step 9: **Slope distance, S, between rows of baffle blocks:**

$S = 2*h_b$ = 6.00 Max S = 6

Use S = 6.00

Step 10: **Minimum depth of cover, j, at outlet to insure that the last row of baffle blocks will be covered by backfill, placed in the structure to the elevation of the downstream grade:**

θ	S	S_y=S*Sinθ	$h_y=h_b$*Cosθ	J = $S_y + h_y$	S_x=S*Cosθ
26.57	6.00	2.68	2.55	5.23	5.37

Step 11: **Apron lengths, L_3 and L_s :**

Drop, F	S	e	J	L_y = e + F + J	Min Rows of blocks=L_y/S_y	L_s=4*S	L_y=4*S_y	L_3=4*S_x	L=$L_1+L_2+L_3$
6.00	6.00	0.02	5.23	11.25	4.00	24.00	10.73	21.47	30.21

Step 12: **Wall heights :**

$h_1=d_1$ + 1ft	$h_2=h_1 - h_s$	$h_3 = 3*h_b$
2.50	2.17	8.32

Step 13: **Determine length, M1, of the upstream wingwalls:**

Input (From Fig 7.2 page 337)

C_1	Diameter of Pipe, D (ft)	Depth of water in canal, d_1(ft)	D/d	$M_1=1.5h_1 + C_1$
2.50	8.00	3.70	2.16	2.16

Step 14: **Determine length, M3, of the downstream wingwalls:**

Input (From Fig 7.2 page 337)

C_3	$h_3{}^1$	$M_3=1.5h_3{}^1 + C_3$
2.50	9.31	16.46

Figure A.26 – Baffled Apron Drop Design Spreadsheet for Conduit Outlet

A.8 Total Dynamic Head Calculations

A hydraulic analysis was performed to calculate the total dynamic head (or maximum pumping head) that the pumps must be able to operate against. The total dynamic head includes static head, pipe friction head losses, and minor head losses from valves and fittings. Two cases were analyzed at each integrated facility location. Case 1 is for diversions of water from the river to the reservoir and Case 2 is for releases of water from the reservoir to the river. Each case results in a different pumping head due to the difference in water levels between the river and the reservoir. The case resulting in the largest pumping head required was chosen as the controlling case in the pump selection. For the Webb Tract at San Joaquin River integrated facility pumping plant, the inputs and assumptions used in the analysis are summarized in Table A.1, and head losses and total dynamic head calculations are given in Tables A.2 and A.3. Similar calculations were performed for each of the other three integrated facility locations.

Table A.1: Inputs and Assumptions Used to Calculate Total Dynamic Head for the Webb Tract at San Joaquin River Integrated Facility Pumps

	Input Description	Input Value
Water Levels:	Minimum River Level	-1.714
	Maximum River Level (100-year)	6.826
	Minimum Submergence Level	-12
	Minimum Reservoir Level	-18
	Maximum Reservoir Level	4
	Bottom of Midbay Pool	-26
Pipe Diameters & Lengths:	Steel Pipe Diameter-Large	8
	Concrete Conduit Pipe Diameter	8
	Length of Steel Pipe CASE 1	50
	Length of Concrete Pipe CASE 1	250
	Length of Steel Pipe CASE 2	50
	Length of Concrete Pipe CASE 2	250
Loss Coefficients:	Formed Suction Intake	0.15
	90 Degree Bend	0.4
	Tee at Conduit Connection	1.5
	Butterfly Valve	0.3
	Exit at Reservoir	1.0
	Darcy Friction Factor (Steel)	0.012
	Darcy Friction Factor (Concrete)	0.014
Pump Information:	Suction Flange Diameter Below the Impeller	8
	Pump Efficiency	0.90
	Motor Efficiency	0.97
	Overall Efficiency	0.87

Table A.2: Case 1 and Case 2 Head Losses and Total Dynamic Head for the Webb Tract at San Joaquin River Integrated Facility 400 cfs Pumps

CASE 1 - Diversion of Water from River to Reservoir											
			Losses								
Flow	Velocity Head (Steel Pipe)	Velocity Head (Conc. Pipe)	FSI	90 Deg. Bend (2)	Tee at Connection to Conduit	Butterfly Valve	Pipe Friction (Steel Pipe)	Pipe Friction (Conc. Pipe)	Exit at Reservoir	**Total Head Loss**	**Total Dynamic Head**
(cfs)	($v_s^2/2g$)	($v_c^2/2g$)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.7
50	0.02	0.02	0.00	0.01	0.02	0.00	0.00	0.01	0.02	0.07	5.8
100	0.06	0.06	0.01	0.05	0.09	0.02	0.00	0.03	0.06	0.26	6.0
150	0.14	0.14	0.02	0.11	0.21	0.04	0.01	0.06	0.14	0.59	6.3
200	0.24	0.24	0.04	0.20	0.37	0.07	0.02	0.11	0.24	1.04	6.8
250	0.38	0.38	0.06	0.31	0.57	0.11	0.03	0.17	0.38	1.63	7.3
300	0.55	0.55	0.08	0.44	0.82	0.16	0.04	0.24	0.55	2.34	8.1
350	0.75	0.75	0.11	0.60	1.12	0.22	0.06	0.33	0.75	3.19	8.9
400	0.98	0.98	0.15	0.78	1.47	0.29	0.07	0.43	0.98	4.16	9.9

CASE 2 - Release of Water from Reservoir to River											
			Losses								
Flow	Velocity Head (Steel Pipe)	Velocity Head (Conc. Pipe)	FSI	90 Deg. Bend (2)	Tee at Connection to Conduit	Butterfly Valve	Pipe Friction (Steel Pipe)	Pipe Friction (Conc. Pipe)	Exit at Reservoir	**Total Head Loss**	**Total Dynamic Head**
(cfs)	($v_s^2/2g$)	($v_c^2/2g$)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	18.8
50	0.02	0.02	0.00	0.01	0.02	0.00	0.00	0.01	0.02	0.07	18.9
100	0.06	0.06	0.01	0.05	0.09	0.02	0.00	0.03	0.06	0.26	19.1
150	0.14	0.14	0.02	0.11	0.21	0.04	0.01	0.06	0.14	0.59	19.4
200	0.24	0.24	0.04	0.20	0.37	0.07	0.02	0.11	0.24	1.04	19.9
250	0.38	0.38	0.06	0.31	0.57	0.11	0.03	0.17	0.38	1.63	20.5
300	0.55	0.55	0.08	0.44	0.82	0.16	0.04	0.24	0.55	2.34	21.2
350	0.75	0.75	0.11	0.60	1.12	0.22	0.06	0.33	0.75	3.19	22.0
400	0.98	0.98	0.15	0.78	1.47	0.29	0.07	0.43	0.98	4.16	23.0

Table A.3: Case 1 and Case 2 Head Losses and Total Dynamic Head for the Webb Tract at San Joaquin River Integrated Facility 150 cfs Pumps

CASE 1 - Diversion of Water from River to Reservoir											
			Losses								
Flow	Velocity Head (Steel Pipe)	Velocity Head (Conc. Pipe)	FSI	90 Deg. Bend (2)	Tee at Connection to Conduit	Butterfly Valve	Pipe Friction (Steel Pipe)	Pipe Friction (Conc. Pipe)	Exit at Reservoir	**Total Head Loss**	**Total Dynamic Head**
(cfs)	($v_s^2/2g$)	($v_c^2/2g$)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.7
25	0.06	0.06	0.01	0.05	0.09	0.02	0.01	0.05	0.06	0.29	6.0
50	0.24	0.24	0.04	0.20	0.37	0.07	0.04	0.21	0.24	1.17	6.9
75	0.55	0.55	0.08	0.44	0.82	0.16	0.08	0.48	0.55	2.62	8.3
100	0.98	0.98	0.15	0.78	1.47	0.29	0.15	0.85	0.98	4.67	10.4
125	1.53	1.53	0.23	1.22	2.29	0.46	0.23	1.34	1.53	7.29	13.0
150	2.20	2.20	0.33	1.76	3.30	0.66	0.33	1.92	2.20	10.5	16.2

CASE 2 - Release of Water from Reservoir to River											
			Losses								
Flow	Velocity Head (Steel Pipe)	Velocity Head (Conc. Pipe)	FSI	90 Deg. Bend (2)	Tee at Connection to Conduit	Butterfly Valve	Pipe Friction (Steel Pipe)	Pipe Friction (Conc. Pipe)	Exit at Reservoir	**Total Head Loss**	**Total Dynamic Head**
(cfs)	($v_s^2/2g$)	($v_c^2/2g$)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	24.8
25	0.06	0.06	0.01	0.05	0.09	0.02	0.01	0.05	0.06	0.29	25.1
50	0.24	0.24	0.04	0.20	0.37	0.07	0.04	0.21	0.24	1.17	26.0
75	0.55	0.55	0.08	0.44	0.82	0.16	0.08	0.48	0.55	2.62	27.5
100	0.98	0.98	0.15	0.78	1.47	0.29	0.15	0.85	0.98	4.67	29.5
125	1.53	1.53	0.23	1.22	2.29	0.46	0.23	1.34	1.53	7.29	32.1
150	2.20	2.20	0.33	1.76	3.30	0.66	0.33	1.92	2.20	10.5	35.3

A.9 Bypass Channel Velocity Profiles

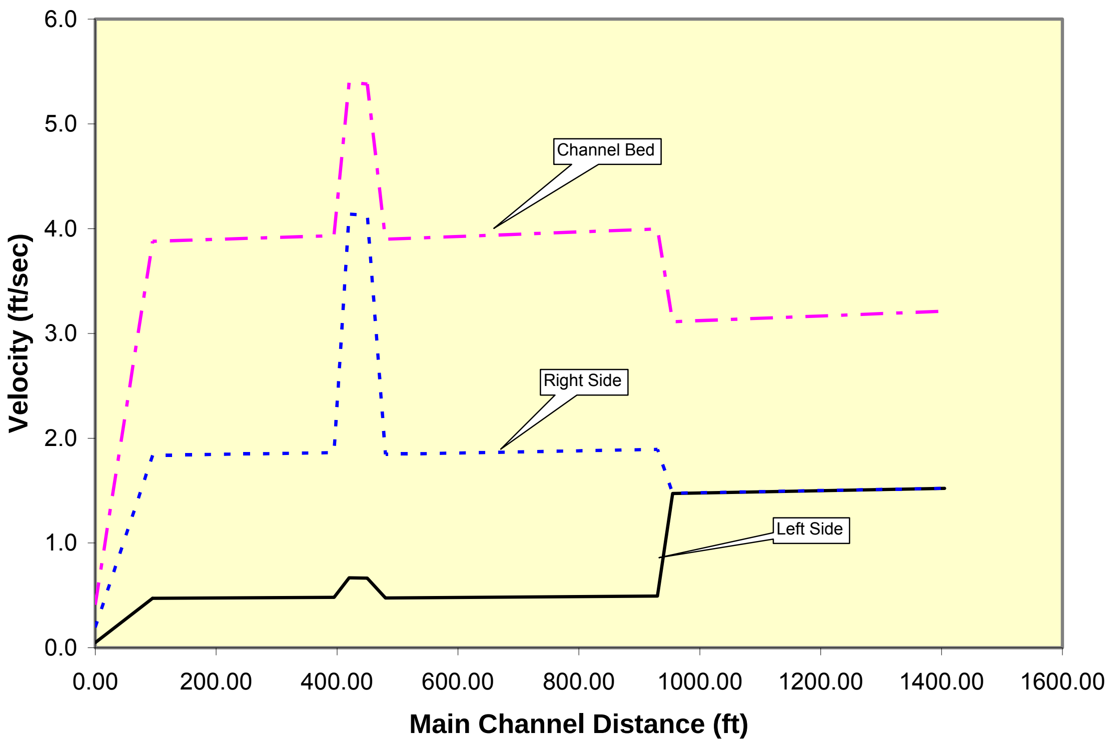

Figure A.27 – Velocity Profile for Bypass Channel at Webb Tract (San Joaquin River and False River Facilities)

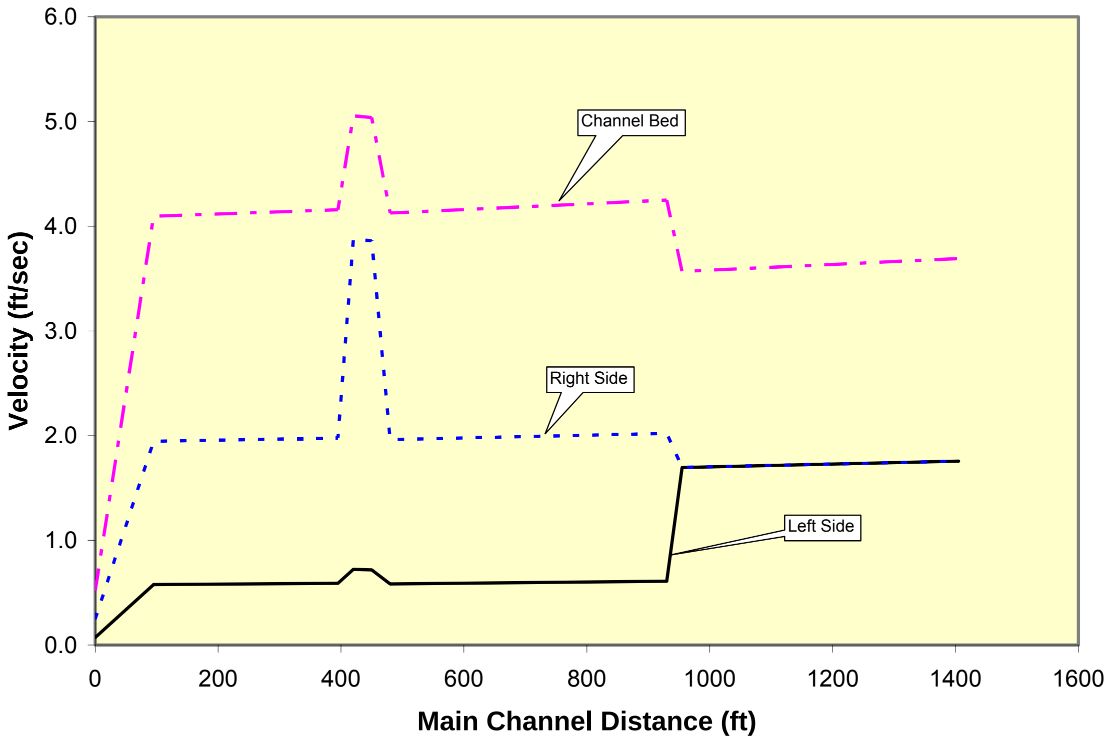

Figure A.28 – Velocity Profile for Bypass Channel at Bacon Island, Middle River

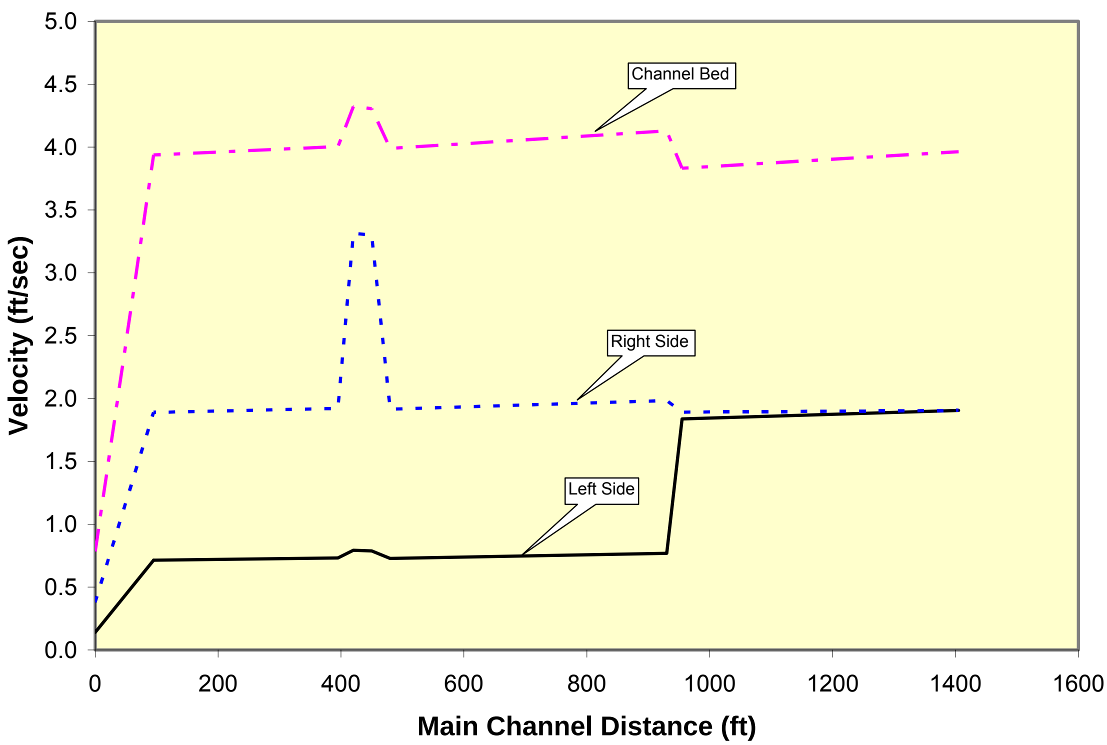

Figure A.29 – Velocity Profile for Bypass Channel at Bacon Island, Santa Fe Cut